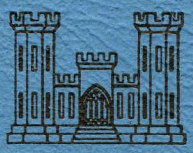
REVIEW OF SOILS DESIGN, CONSTRUCTION AND PERFORMANCE OBSERVATIONS TOM JENKINS DAM, OHIO



TECHNICAL REPORT NO. 3-474

March 1958

U. S. Army Engineer Waterways Experiment Station
CORPS OF ENGINEERS
Vicksburg, Mississippi

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ARMY-MRC VICKSBURG, MISS.

Frontispiece. Aerial view of Tom Jenkins Dam

PREFACE

This report is one of a number of reports describing the construction and behavior, from the standpoint of foundations and soil mechanics, of earth dams completed by the Corps of Engineers since 1950. These reports are prepared by the Waterways Experiment Station as part of the Civil Works Investigations program of the Office, Chief of Engineers, under CWI Item 505, "Prototype Analysis (Soil)." The purpose of these studies is to compare the performance of the completed structure with design predictions, and by these comparisons provide information that may be valuable in the design and construction of future projects by the Corps of Engineers.

This report is a review of the soils design, construction, and performance of Tom Jenkins Dam, formerly Burr Oak Dam. The dam was designed by and built under the supervision of the U. S. Army Engineer District, Huntington, W. Va., and data on the design and construction and prototype observations were furnished by that district. Design details not included may be found in "Definite Project Report on Burr Oak Reservoir, East Branch of Sunday Creek, Ohio," issued by the Huntington District in September 1947.

This report was prepared by Mr. R. C. Sloan under the direction of Messrs. W. J. Turnbull, W. G. Shockley, and R. W. Cunny of the Soils Division, Waterways Experiment Station. It was reviewed prior to publication by personnel of the Huntington District and Ohio River Division, and was reviewed and approved by the Office, Chief of Engineers.

CONTENTS

	Page
PREFACE	iii
NOTATIONS	vii
SUMMARY	ix
PART I: GENERAL INFORMATION	1
PART II: INVESTIGATIONS AND DESIGN ANALYSES	6
Foundation	6
Embankment	17
Spillway and Outlet Works	24
PART III: CONSTRUCTION	25
Borrow Areas and Materials	25
Embankment	25
PART IV: PERFORMANCE OBSERVATIONS	34

List of Tables

Table	Title	Page
1	Pertinent Dam and Reservoir Data	3
2	Summary of Test Data for Foundation Soils	10 & 11
3	Summary of Test Data on Borrow Materials	19
4	Soils Design Constants	20
5	Summary of Field Compaction Control Tests	30
6	Summary of Test Data for Embankment Record Sample	31

List of Illustrations

Figure		
	Frontispiece. Aerial view of Tom Jenkins Dam	ii
1	General plan and vicinity map	2
2	Center-line profile showing limits of grouting and sand and gravel blanket, settlement plates, and first construction stage	4
3	Selected logs of borings along center line of dam	7 & 8
4	Locations of borings, test pits, and settlement plates	9
5	Gradation of typical foundation soils	12
6	Consolidated-drained direct shear strength curves for foundation soils	12
7	Consolidation curves for foundation soils	14
8	Elevation of fill and predicted and observed settlements for plates 1, 2, 4, and 8	16
9	Gradation of composite borrow materials	18
10	Consolidated-drained direct shear strength curves for borrow materials	20
11	Typical embankment sections	21
12	Gradation bands for filter and drainage blanket sand and gravel	23
13	Field control curves and test results	27
14	Variation from optimum water content and maximum standard density for field control tests	29
15	Gradation of record samples	30
16	Consolidation curves for record samples	32
17	Consolidated-drained direct shear strength curves for record samples	33
18	Settlement plate detail	34

NOTATIONS

b	One-half base width of embankment
c	Cohesion, tons per sq ft
e	Void ratio
e_o	Initial void ratio
h	Head of water
k	Coefficient of permeability
L	Distance from center of embankment to downstream toe
LL	Atterberg liquid limit
p	Pressure, tons per sq ft
PI	Atterberg plasticity index (LL-PL)
PL	Atterberg plastic limit
Q	Seepage, gal per min
S	Degree of saturation
T	Depth of overburden to bedrock
w	Water content, per cent of dry weight
γ_d	Unit dry weight, lb per cu ft
γ_m	Unit moist weight, lb per cu ft
γ'	Unit submerged weight, lb per cu ft
ϕ	Angle of internal friction, degrees
UU	Unconsolidated-undrained shear test
CU	Consolidated-undrained shear test
CD	Consolidated-drained shear test

SUMMARY

The dam for the multipurpose Tom Jenkins Reservoir Project is a rolled-earth fill structure about 75 ft high, 944 ft long, containing about a half million cubic yards of earth fill. Construction was begun in March 1948 and completed in February 1950. A gate-controlled outlet structure consisting of a single 8-ft-diameter horseshoe-shaped conduit with a capacity of 1380 cfs with the reservoir pool at top of water-supply pool is located in one abutment, and an unlined, uncontrolled spillway is located in a saddle about 1000 ft upstream of the outlet works. The maximum spillway capacity is 21,700 cfs with a surcharge head of 20 ft above the concrete weir crest. Water-supply pool and maximum design pool elevations will create net heads of 30 and 70 ft, respectively, above the valley floor.

Foundation

The alluvial deposit in the valley varies in depth from 25 to 35 ft, the upper 12 to 20 ft of which is relatively impervious silts, clays, and sands, and the lower portion is semipervious sands, silts, and gravels. A design shearing strength of $\phi = 29^{\circ}$ and $c = 0.05$ ton per sq ft was used for the upper foundation soils. The lower sand and gravel stratum was assumed to have the same strength for design purposes. The alluvial deposits are underlain by bedrock of alternating layers of sandstone and sandy shale.

The coefficient of permeability indicated by laboratory tests performed on undisturbed samples ranged from 0.296×10^{-4} to 0.00046×10^{-4} cm per sec for the upper impervious layer. No permeability tests were performed on the underlying semipervious materials and the coefficient of permeability used for seepage computations is not known.

Embankment

The embankment consists of an unzoned section of relatively impervious materials with upstream and downstream slopes ranging from 1 on 2.5 near the top to 1 on 4.5 near the base. The embankment was constructed of alternate wet and dry layers placed in 7- to 9-in. loose lifts and compacted with 9 to 15 passes of a sheepsfoot roller exerting foot pressures of 355 psi. A field control testing program conducted during construction indicated the density achieved averaged about 100 per cent of standard compaction. However, laboratory density tests performed on undisturbed record samples of the embankment indicated densities ranged from 89 to 101 per cent with an average of only 94 per cent of standard densities. Consolidated-drained direct shear tests performed on four undisturbed record samples indicated average shearing strength values of $\phi = 24^{\circ}$, $c = 0.28$ ton per sq ft as compared with the design value of $\phi = 30^{\circ}$, $c = 0.04$ ton per sq ft.

The upstream slopes are protected by dumped stone from the natural ground to four feet above conservation and water-supply pool elevation. The lower portion of the downstream slope is also protected with dumped stone. Both slopes are protected with sod above the stone to the crest.

Seepage through the embankment is collected by a 3-ft-thick sand and gravel drainage blanket beneath the downstream slope. No seepage has been observed at the downstream toe to date. However, only small reservoir heads have been experienced, the maximum being about 34 ft above the valley flood plain in March 1955.

Spillway and Outlet Works

The spillway and outlet works are founded on firm rock so the foundation for these structures is not discussed in this report.

REVIEW OF SOILS DESIGN, CONSTRUCTION, AND PERFORMANCE OBSERVATIONS

TOM JENKINS DAM, OHIO

PART I: GENERAL INFORMATION

1. The Tom Jenkins Reservoir Project is a multipurpose project providing means for flood control, water supply, and recreation. The embankment for the reservoir (formerly called Burr Oak Dam) is located about 1/4 mile above the mouth of the East Branch of Sunday Creek, a tributary of Hocking River, and about three miles north of Glouster and four miles southeast of Corning, Ohio. The frontispiece is a general view of the embankment and reservoir; a vicinity map is included in fig. 1.

2. The dam is a rolled-fill earth structure, 944 ft in length at the crest elevation of 765,* and has a height above the valley floor of about 75 ft with a maximum height of 84 ft above the stream bed. The crown is 23 ft wide and supports a bituminous surfaced roadway. The side slopes average about 1 on 3-1/2. The upstream slopes are protected by dumped stone from the ground to four feet above conservation and water-supply pool el 721; from the upper limit of the stones to the top of the dam, the slopes were provided with topsoil to support a grass cover. The downstream slopes from the ground surface to el 708 are protected from seepage and tailwater by dumped stone, and from el 708 to the top of the dam the slopes are sodded. The embankment has a total volume of 517,000 cu yd. A plan of the dam and surrounding area is shown in fig. 1.

3. The outlet works, located in the south (left) abutment, consist of the usual appurtenances and a single 8-ft-diameter, horseshoe-shaped conduit tunneled through rock and indurated clay and lined with reinforced concrete. The tunnel has a maximum capacity of 1380 cfs with the reservoir pool at el 721, and flow is controlled by three 3.5- by 7-ft service gates. The spillway, located in a saddle of a ridge about 1000 ft south of the outlet works, is an uncontrolled, unlined chute type with a maximum design capacity of 21,700 cfs at a surcharge head of 20 ft above the crest at el 740. Pertinent dam and reservoir data are given in table 1.

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* All elevations are in feet above mean sea level.

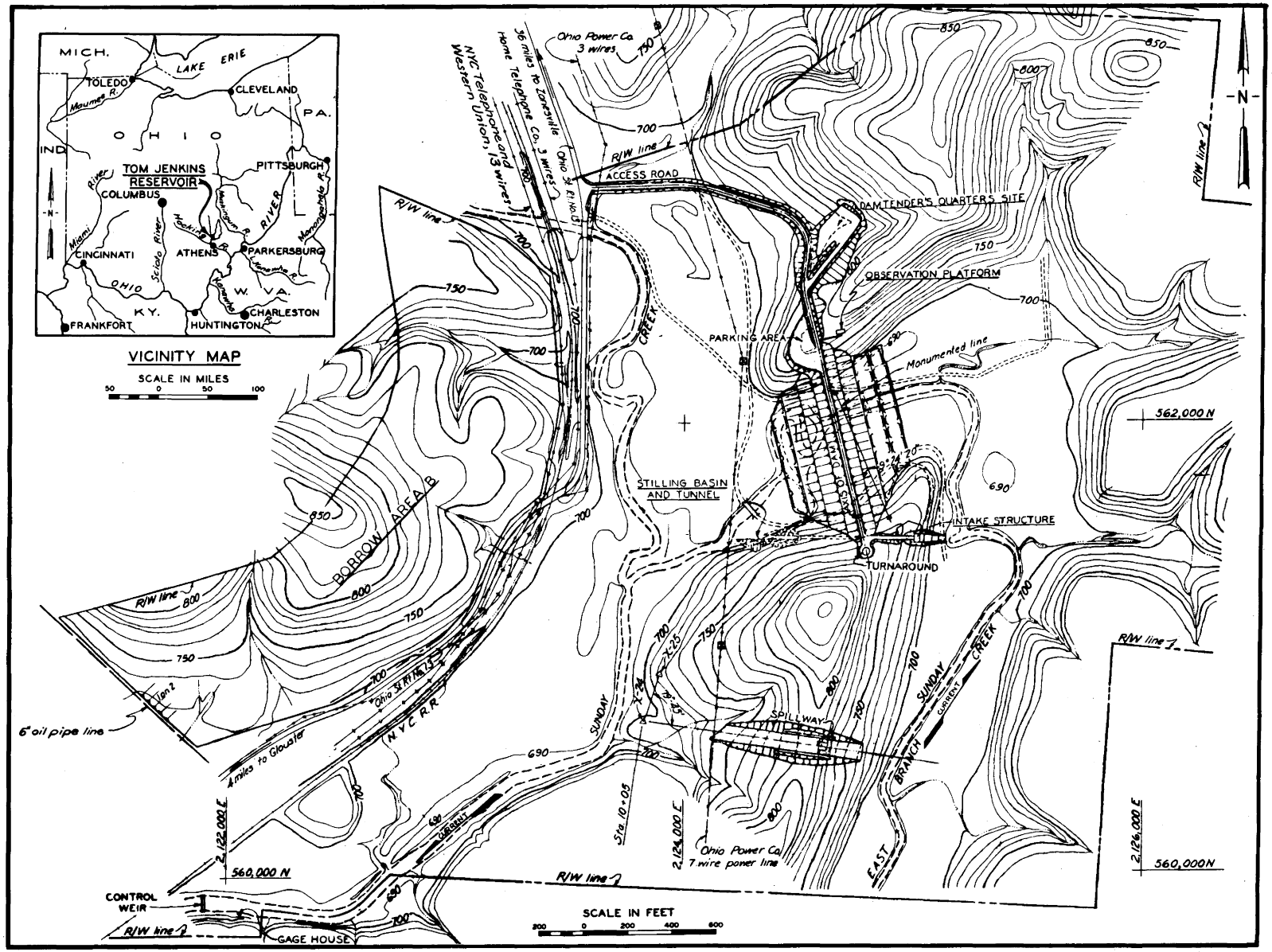


Fig. 1. General plan and vicinity map

Table 1

Pertinent Dam and Reservoir Data

Drainage area	32.8 sq mi
Maximum discharge of record	700 cfs
Minimum conservation pool el	710.0
Maximum conservation pool el	721.0
Maximum design flood pool el	760.0
Embankment	
Crest el	765.0
Length	944.0 ft
Height (maximum above flood plain)	84.0 ft
Volume	517,000 cu yd
Spillway	
Crest length	80.0 ft
Crest el	740.0
Discharge at maximum design flood pool	21,700 cfs
Outlet works	
Conduit (horseshoe-shaped) diameter	8.0 ft
Number of gates	3
Size of gates	3.5 x 7 ft
Intake invert el	688.5
Capacity at maximum conservation pool	1,380 cfs
Stilling basin floor el	680.0
Stilling basin end sill el	682.0

4. The project was constructed under two contracts. Construction of the outlet works tunnel and stilling basin and the excavation for the intake control structure were performed under the first contract; this work was begun in March 1948 and completed in April 1949. The embankment, spillway, and the outlet works intake control tower were constructed under a second contract which was begun in September 1948 and completed in February 1950.

5. The embankment between sta 5+00 and the north (right) abutment was constructed in two stages to allow anticipated pore pressures in the relatively thick and impervious foundation overburden material to partially dissipate before the total fill load was placed. The first construction stage extended from October to December 1948 and the second stage from May to September 1949. The extent of the work accomplished on the embankment during the first construction stage is shown in fig. 2. Pore pressures

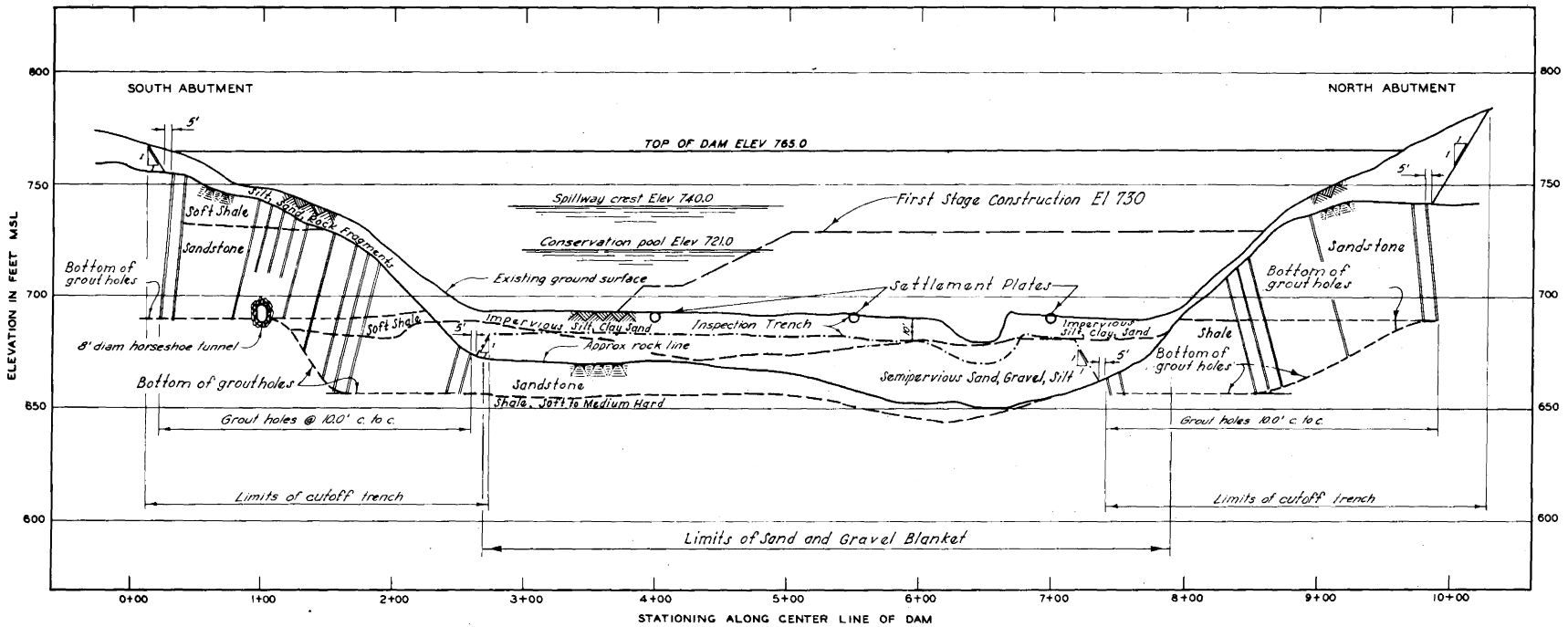


Fig. 2. Center-line profile showing limits of grouting and sand and gravel blanket, settlement plates, and first construction stage

beneath the rest of the embankment, where the impervious stratum was less thick, were not predicted to be as great and this portion was built in one construction season.

6. Water was impounded in the reservoir during February 1952; the highest reservoir stage to date (el 724.5) occurred in March 1955.

PART II: INVESTIGATIONS AND DESIGN ANALYSES

Foundation

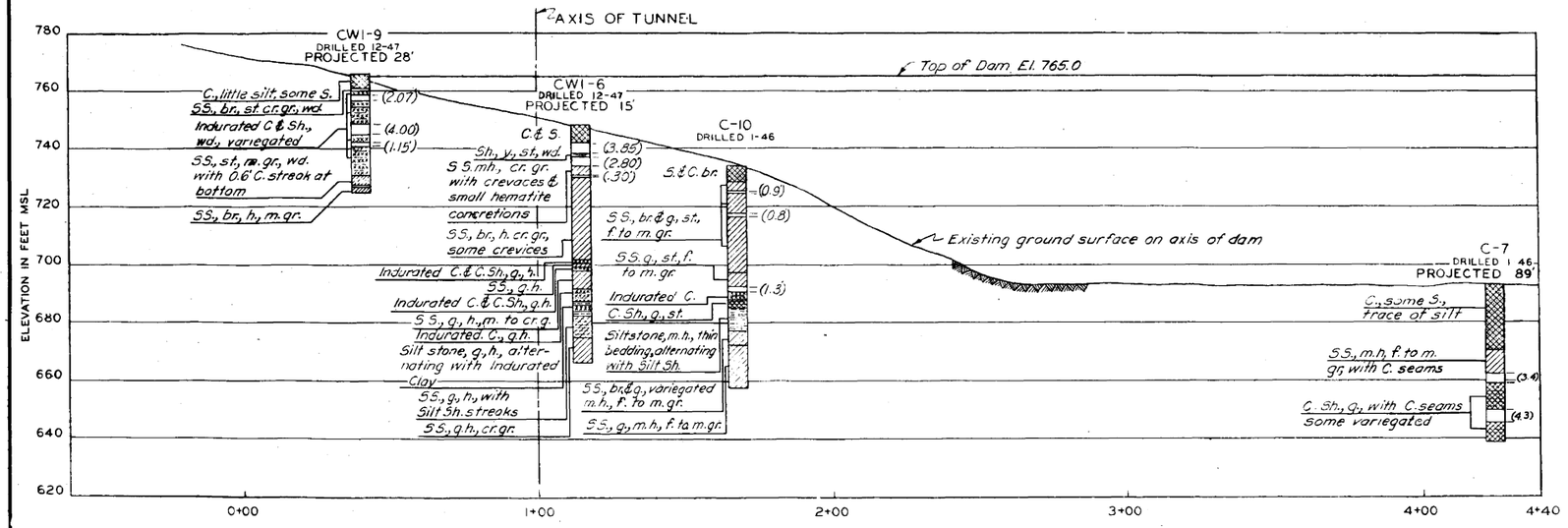
Geology

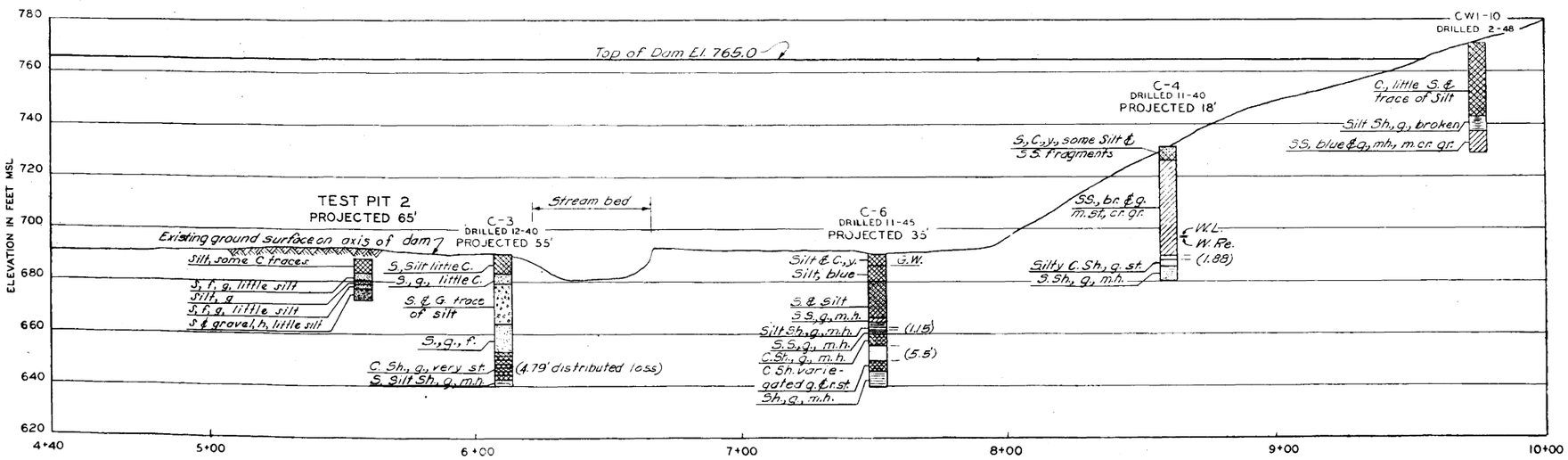
7. The dam site is located in an unglaciated portion of the Appalachian physiographic province. The stream at the dam site flows westward along the north, or right, side of the valley. From the flood plain, which is 10 ft above low water elevation of 681, the north abutment rises steeply to el 820 and the south, or left, abutment rises on a moderately steep slope to el 840. The width of the valley at flood plain level is about 530 ft and at the elevation of the top of the dam is approximately 950 ft. The rocks in the area have been downfolded into the Appalachian geosyncline. As the area lies along the westward limb of the structural trough, the strata have a gentle southeastward dip. No indications of major faulting were found in the area although joint fractures, which resulted from stresses induced by uplift, are very numerous.

8. The foundation overburden consists of two general groups: alluvium on the valley flood plain, and residual soil on the valley slopes. The flood plain alluvium occurs in two fairly distinct strata. A relatively impervious stratum of silt, sand, and clay ranging from 12 to 20 ft in thickness overlies a semipervious sand, silt, and gravel stratum ranging from 5 to 25 ft in thickness. Residual soils on the valley slopes were developed from sandstone and shale and are composed of silt, sand, and clay. These soils range from 4 to 22 ft in thickness. The overburden soils are underlain by bedrock of alternating strata of shale and sandstone at depths of 20 to 40 ft beneath the valley floor. The bedrock becomes exposed, or generally lies near the surface, in both abutments. The depth of weathering of the bedrock under the dam site varies from about 2 to 10 ft.

Field exploration

9. The foundation exploration program consisted of undisturbed sampling, core-drilled and auger borings, and one test pit (TP 2). Locations of selected foundation borings and the test pit are shown in plan in fig. 4, page 9. A generalized soil profile of the foundation along the center line of the embankment is shown in fig. 2 and logs of borings in fig. 3.





LEGEND

- C. (Clay) Silt
- S. (Sand)
- G (Gravel)
- Rock Frag. (fragments)
- S. & C., S. & Silt or S. & Lm. (any two soil sym may be combined)
- S.S. (Sandstone, Unweathered)
- S.S. (Sandstone, Weathered)
- Sh. (Shale)
- C. Sh. or C. & Sh. interbedded
- Silt Sh.
- S Sh (Sandy Sh.)
- Siltstone
- Indurated Clay
- Coal
- (5.5) Core loss

Abbreviations

brown	br
coarse	cr
distributed	dist.
fine	f
gray	g
grained	gr
Ground Water	G W
hard	h
medium	m
red	r
soft	st
variegated	v
weathered	wd
yellow	y
Water Loss	WL
Water Regained	W Re

NOTES

FOR PLAN OF FOUNDATION EXPLORATION AND LOCATIONS OF TEST BORINGS, SEE FIG. 4.

FOR GENERALIZED SOIL PROFILE, SEE FIG. 2.

Fig. 3. Selected logs of borings along center line of dam

10. Samples from the core-drilled holes in the abutment rocks indicated the possibility of detrimental seepage occurring through the jointed sandstone; however, no tests were performed to estimate the quantity of seepage that could be expected.

Laboratory tests

11. Visual classifications were made of samples obtained from the auger and core-drilled holes and the test pit. Water contents and mechanical analyses were not performed on the auger hole samples because inspection of these samples was delayed six years by postponement of design activity for this project during World War II. Mechanical analyses, Atterberg limits, specific gravity, water content, density, consolidated-drained direct shear, permeability, and consolidation tests were performed on undisturbed samples from the core-drilled holes and test pit 2. The samples from the undisturbed sample borings DU-3, -4, -6, -7, and -8, and test pit 2 were considered representative of the foundation materials and results of laboratory tests on these samples were used to determine the design values for the foundation soils.

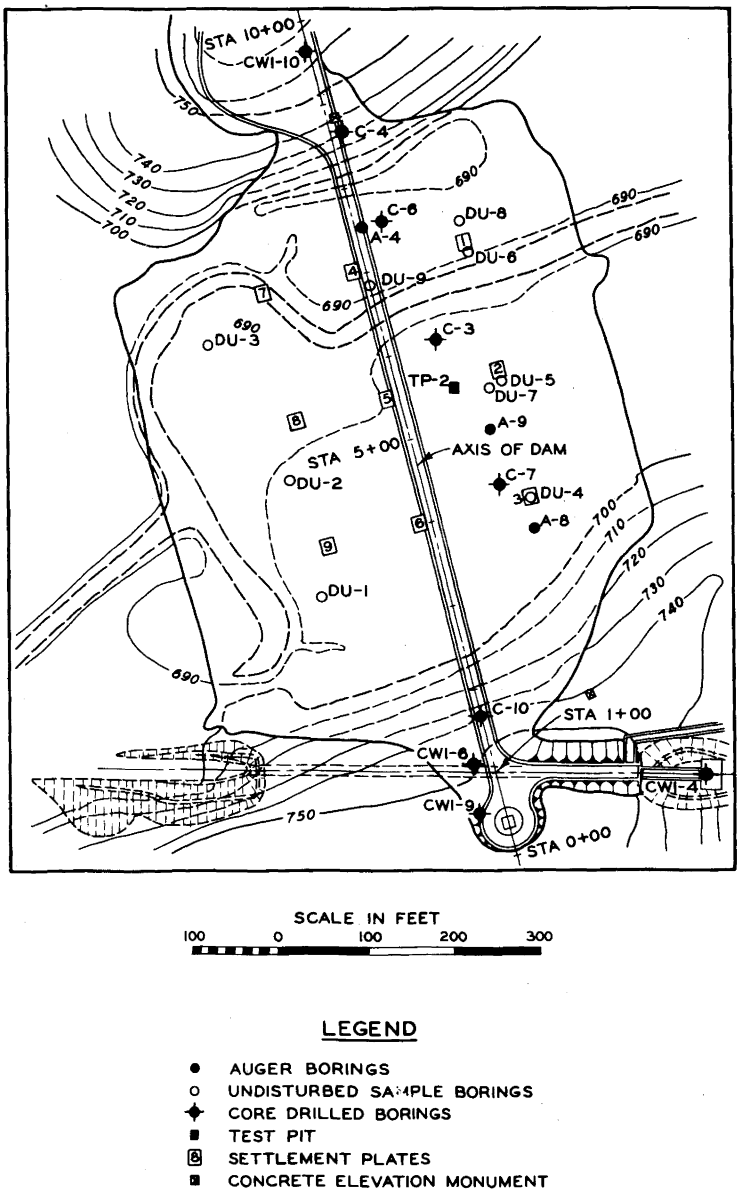


Fig. 4. Locations of borings, test pits, and settlement plates

12. Results of mechanical analyses, Atterberg limits, specific gravity, and natural water content and density tests are shown in table 2,

TABLE 2. SUMMARY OF TEST DATA FOR FOUNDATION SOILS

REF NO.	TEST PIT OR BORING NO.	SAMPLE	SAMPLE DEPTH OR EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS*				ATTER-BERG LIMITS		SP GR	NAT w %	NAT DRY DENSITY LB/CU FT	COMPACTION		SHEAR DATA											PERMEABILITY		CONSOLIDATION DATA				REMARKS
					GVL %	SAND %	FINES %	D_{10} MM	LL	PL				OPT w %	MAX DRY DENSITY LB/CU FT	INI-TIAL e	DRY DENSITY LB/CU FT	w_i %	AVG w_f %	S_i %	TYPE TEST**	SPECIMEN SIZE IN.	SPECIMEN NO.	σ_{III} TONS/SQ FT	σ_i TONS/SQ FT	c TONS/SQ FT	ϕ DEG	LOAD TONS/SQ FT	k CM/SEC X 10^{-4}	P_o TONS/SQ FT	P_c TONS/SQ FT	C_c	t_{50} MIN	
1	TP-2	1	687.8		--	7	93	----	42	27	2.71	31.9	82.2														0.5	0.00036	0.96	0.29			
																											1.0	0.00046					
2		2	686.1		--	29	71	----	--	--		----															0.0	1.96		----	----		
3		3	685.2		--	17	83	0.001	30	22	2.71	26.2	93.8					23		CD-DS					0.06	30	0.504	0.0065		1.72	0.14		
																											1.007	0.0047					
4		4	683.3		--	25	75	.001	28	22	2.72	20.6	89.3					25		CD-DS					0.03	28	0.504	0.145		2.65	0.25		
																											1.007	0.115					
5		5	681.7		--	86	14	-----	--	--		----															0.0	15.9					
6		6	680.7		--	22	78		27	20	2.70	27.9	94.8					22		CD-DS					0.20	29	0.500	0.0047		2.00	0.17		
																											1.00	0.0044					
7		7	678.3		--	60	40	.005	--	--	2.70	24.5	97.3					23		CD-DS					0.0	31.5	0.508	0.296		2.25	0.16		
																											1.015	0.254					
8		8	677.1		--	42	58	.0015	--	--		--															0.0	0.100					
9	DU-3	1	686.6		--	17	83	-----	--	--		32.9	84.6					26		CD-DS					0.14	26.5	0.25	0.00146		1.50	0.28		
																											4.06	0.00021					
10		2	680.6		--	80	20	-----	--	--		23.5	97.2					25		CD-DS					0.03	31	0.25	0.0380		4.60	0.26		
																											4.06	0.0133					
11		3	674.1		--	38	62	-----	--	--		14.5	117.9					14		CD-DS					0.06	31	0.25	0.00057		3.80	0.09		
																											4.06	0.00014					
12	DU-4	1	690.5		--	0	100	-----	--	--		24.5	100.3					24		CD-DS					0.28	27	0.25	0.00032		1.40	0.08		
																											4.03	0.00013					
13		2	683.5		--	32	68	-----	--	--		30.5	90.3					26		CD-DS					0.02	30.5	0.25	0.00076		2.4	0.24		
																											4.06	0.00038					
14	DU-6	1	687.1		--	29	61	.0015	--	--		25.0	97.2					23		CD-DS					0.23	25	0.25	0.0034		1.7	0.09		
																											4.06	0.00121					
15		2	684.6		--	25	75	-----	--	--		33.7	86.8														0.25	0.0053		1.4	0.22		
																											4.03	0.00107					
16		3	675.6		--	29	61	-----	--	--		26.7	96.8					24		CD-DS					0.06	28	0.51	0.00074		4.4	0.45		
																											4.06	0.00034					
17		4	672.1		--	12	88	.001	--	--		20.1	107.5					18		CD-DS					0.30	23	0.25	0.00098		0.90	0.12		
																											4.06	0.00022					
18		5	669.6		--	--	--	-----	--	--		16.9	115.5					18		CD-DS					0.45	19.5	0.25	0.00025		1.90	0.10		
																											4.06	0.00029					
19	DU-7	1	688.3		--	39	61	.0012	--	--		26.1	92.7					24		CD-DS					0.0	27	0.25	0.00252		1.3	0.20		
																											4.00	0.00059					
20	DU-7	2	679.8		--	65	35	.004	--	--		54.6	-----					35		CD-DS					0.0	29							
21	DU-8	1	686.5		--	16	84	-----	--	--		25.0	91.3					26		CD-DS							0.51	0.214		2.0	0.23		
																									0.11	20.5	4.03	0.00296					

WES FORM NO. 956
OCTOBER 1956

TABLE 2 (CONTINUED)

REF NO.	TEST PIT OR BORING NO.	SAMPLE	SAMPLE DEPTH OR EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS*				ATTER-BERG LIMITS		SP GR	NAT w	NAT DRY DENSITY	COMPACTION		SHEAR DATA										PERMEABILITY		CONSOLIDATION DATA				REMARKS
					GVL %	SAND %	FINES %	D_{10} MM	LL	PL		%	LB/CU FT	OPT w %	MAX DRY DENSITY LB/CU FT	INI-TIAL e	DRY DENSITY LB/CU FT	w_i %	AVG w_f %	S_i %	TYPE TEST**	SPECIMEN SIZE IN.	SPECIMEN NO.	σ_{1i} TONS/SQ FT	σ_i TONS/SQ FT	c TONS/SQ FT	ϕ DEG	LOAD TONS/SQ FT	k CM/SEC X 10^{-4}	P_o TONS/SQ FT	P_c TONS/SQ FT	C_c	t_{50} MIN	
22	DU-8	2	680.4		--	28	72	.0028	--	--		32.6	88.3						26		CD-DS							0.50	0.00176		2.0	0.22		
																										0.12	27	4.03	0.00124					
23		3	677.4		--	56	44	.0018	--	--		16.3	116.6						15		CD-DS							0.51	0.00027		1.1	0.08		
																												4.06	0.00017					
																										0.12	29							
24		4	674.4		--	45	55	----	--	--		17.6	112.8						16		CD-DS							0.51	0.0001		3.1	0.12		
																										0.02	32	4.06	0.00007					

* Based on MIT classification: sand 2 mm to 0.06 mm; fines include silt and clay size, <0.06 mm.
** Specimens shear at rate of 0.001 in. per min.
CD-DS Consolidated drained-direct shear.

WES FORM NO. 956
OCTOBER 1956

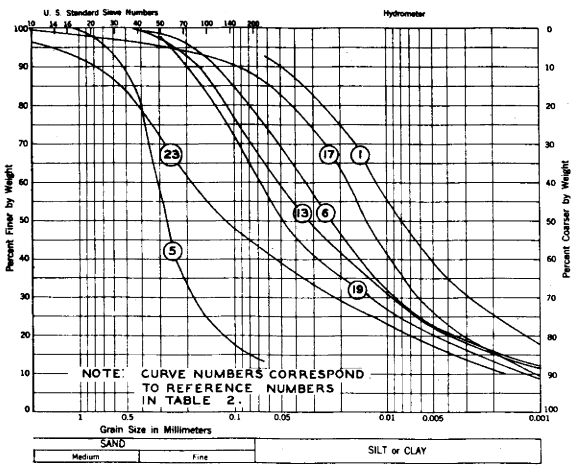
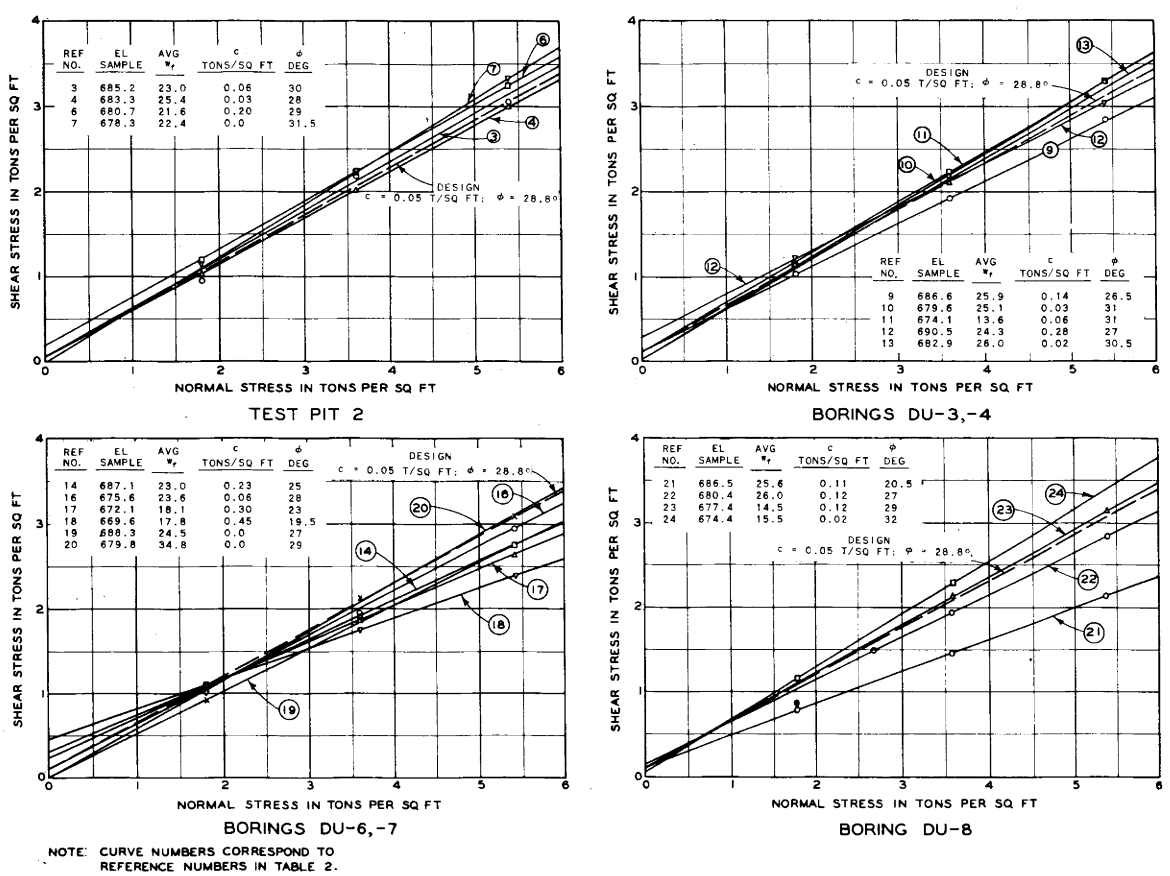


Fig. 5. Gradation of typical foundation soils

together with results of consolidated-drained direct shear, consolidation, and permeability tests performed on materials from the upper impervious stratum. Typical gradation curves of these materials are shown in fig. 5. Tests were not performed on the lower semipervious sands, silts, and gravels.

13. In fig. 6 results are plotted of four consolidated-drained direct shear tests performed on undisturbed samples obtained from test pit 2, together with results of consolidated-drained direct shear tests performed on undisturbed


SHEAR STRESS IN TONS PER SQ FT
NORMAL STRESS IN TONS PER SQ FT
TEST PIT 2
DESIGN c = 0.05 T/SQ FT; φ = 28.8°
BORINGS DU-3,-4
BORINGS DU-6,-7
BORING DU-8
NOTE: CURVE NUMBERS CORRESPOND TO REFERENCE NUMBERS IN TABLE 2.

REF NO.	EL SAMPLE	AVG w_f	c TONS/SQ FT	φ DEG
3	685.2	23.0	0.06	30
4	683.3	25.4	0.03	28
6	680.7	21.6	0.20	29
7	678.3	22.4	0.0	31.5

REF NO.	EL SAMPLE	AVG w_f	c TONS/SQ FT	φ DEG
9	686.6	25.9	0.14	26.5
10	679.6	25.1	0.03	31
11	674.1	13.6	0.06	31
12	690.5	24.3	0.28	27
13	682.9	26.0	0.02	30.5

REF NO.	EL SAMPLE	AVG w_f	c TONS/SQ FT	φ DEG
14	687.1	23.0	0.23	25
16	675.6	23.6	0.06	28
17	672.1	18.1	0.30	23
18	669.6	17.8	0.45	19.5
19	688.3	24.5	0.0	27
20	679.8	34.8	0.0	29

REF NO.	EL SAMPLE	AVG w_f	c TONS/SQ FT	φ DEG
21	686.5	25.6	0.11	20.5
22	680.4	26.0	0.12	27
23	677.4	14.5	0.12	29
24	674.4	15.5	0.02	32

Fig. 6. Consolidated-drained direct shear strength curves for foundation soils

samples from borings DU-3, -4, -6, -7, and -8. The design shearing strength for the foundation of $\phi = 28.8^{\circ}$ and c = 0.05 ton per sq ft was selected on the basis of the tests performed on samples from test pit 2, in which ϕ ranged from 28 to 32° and c from 0.0 to 0.20 ton per sq ft. The selected design values were slightly less than the average of the four. Fifteen similar tests on undisturbed samples from borings DU-3, -4, -6, -7, and -8 were performed to check the validity of the originally selected design shearing strength; in these tests ϕ ranged from 20.5 to 32° and c from 0.0 to 0.45 ton per sq ft.

14. Five consolidation tests were performed on samples from test pit 2. These tests indicated that the relatively impervious soils of the upper stratum would consolidate fairly slowly during construction and excessive pore pressures would develop in the foundation unless the rate of construction was controlled. Results of consolidation tests performed later on undisturbed samples from DU-3, -4, -6, -7, and -8 were considered more applicable for computing the total amounts of foundation settlement and were used to revise the originally computed settlements. The pressure-void ratio curves for both series of tests are shown in fig. 7 and the test results are given in table 2.

15. Eight permeability tests were performed on undisturbed samples of the upper impervious materials from test pit 2. Fifteen permeability tests were also performed on undisturbed samples of the upper stratum from borings DU-3, -4, -6, -7, and -8. Test results indicated that the permeability of the upper stratum of material ranged from about 2.0 to 0.0001 x 10^{-4} cm per sec. Data are not available to indicate whether the tests were performed to determine the horizontal or vertical coefficient of permeability. The results of the permeability tests are included in table 2.

16. No laboratory tests were performed on the underlying bedrock.

Design analyses

17. Shearing stress. The maximum shearing stresses in the foundation overburden material that would be induced by the embankment load were computed by methods presented by Jurgenson.* Shearing resistance that

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* Leo Jurgenson, "The applications of the theories of elasticity and plasticity to foundation problems," Journal of the Boston Society of Civil Engineers (July 1934).

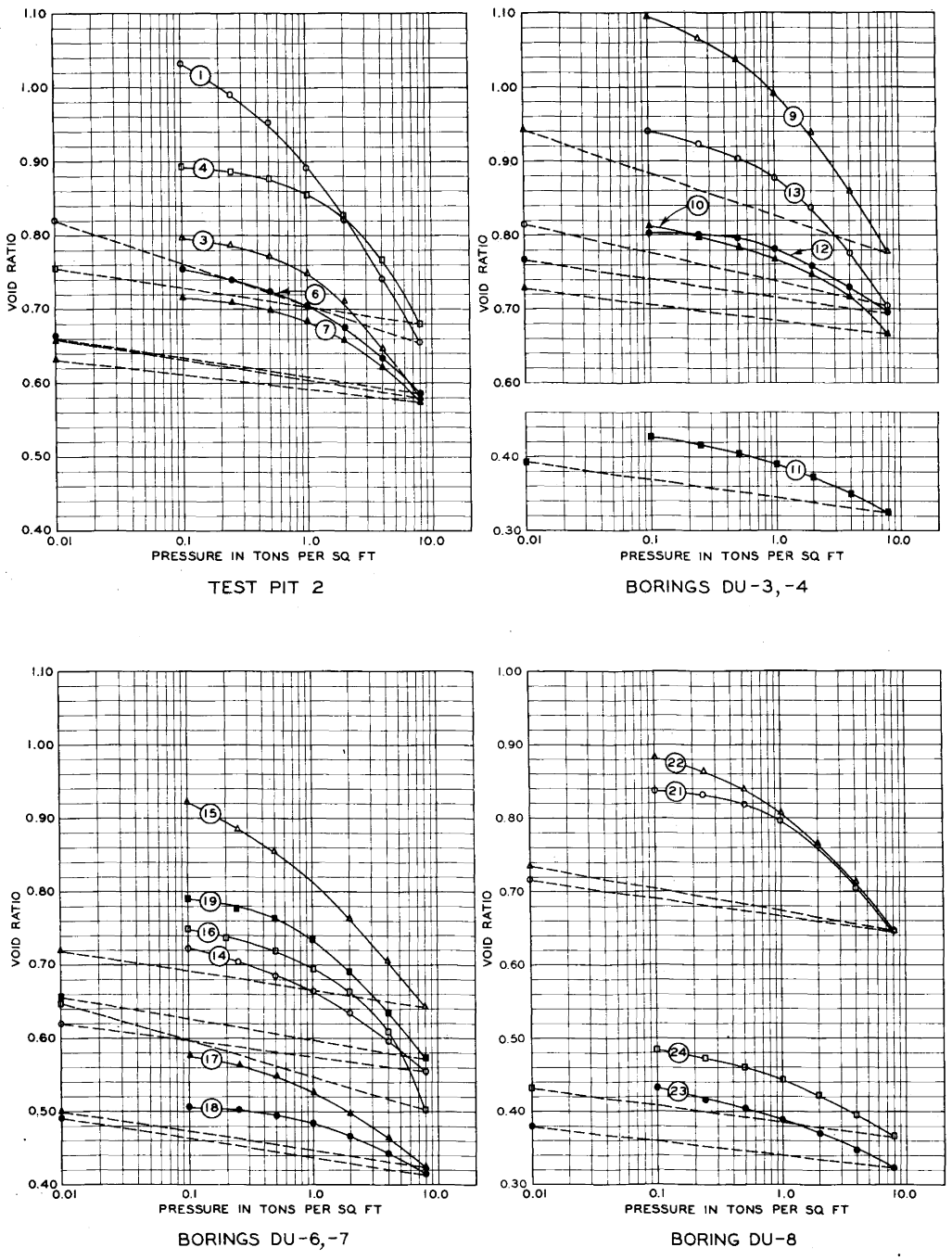
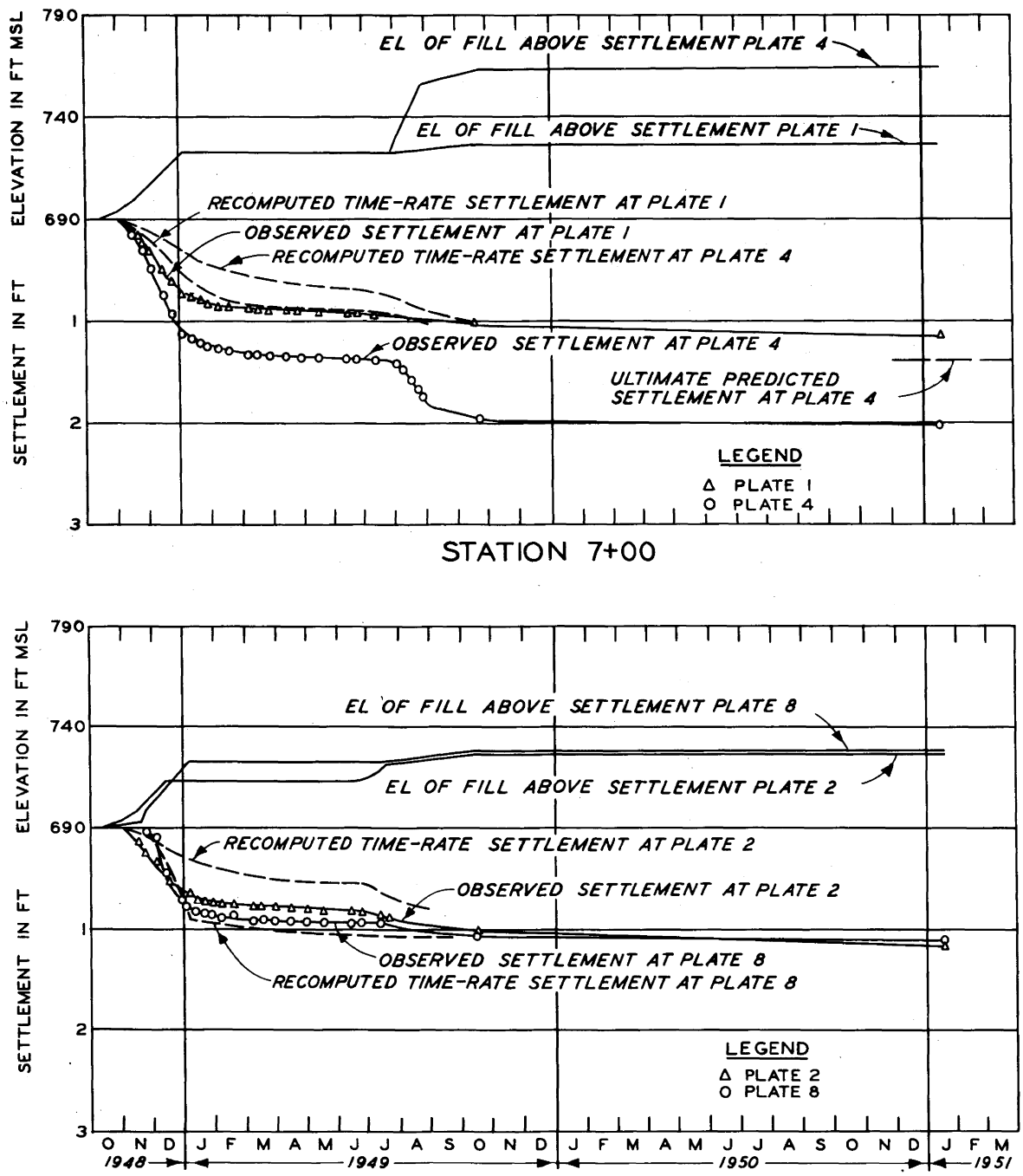


Fig. 7. Consolidation curves for foundation soils

could be mobilized in the foundation soils was computed by Coulomb's equations, assuming the relatively impervious overburden material would have consolidated only 35 per cent for the embankment load immediately after a 90-day construction period. For an embankment section with average slopes of 1 on 3 the minimum computed factor of safety against exceeding the allowable stresses in the foundation was 1.76. On the basis of this, the foundation was believed to be capable of supporting the proposed embankment with average slopes of 1 on 3-3/4.

18. Settlement. Settlement computations performed during the preliminary foundation investigation were based on results of five consolidation tests on samples from test pit 2. For these computations, in which a fill height of 80 ft was used and the foundation was assumed to consist of a 23-ft-thick, relatively impervious top stratum overlying relatively pervious sands and gravels, a total settlement of 2.14 ft was indicated. Subsequent consolidation tests performed on the DU boring samples indicated that the foundation between sta 5+00 and the right abutment could be expected to be only 20 per cent consolidated for the embankment load at the end of a 90-day construction period and about 35 per cent from sta 5+00 to the left abutment. It was believed that if the foundation consolidation were only 20 per cent at the end of construction, the effective intergranular stresses in the foundation would be such that the factor of safety for stability of the embankment would be only about 1.0, and therefore it was specified that the right portion of the embankment be constructed in two construction seasons. Two-season construction would permit additional consolidation and dissipation of excess pore pressure before the full embankment load was placed on the foundation. The factor of safety for the left portion was computed to be adequate (1.25) for one-season construction.

19. Rate of settlement vs time curves were computed for the center line of the embankment based on conditions at holes DU-3, DU-4, and DU-6 (see fig. 4). However, during the early stage of construction nine settlement plates were installed (see locations in fig. 4) and time-rate curves were computed for these using uniform rates of construction and the DU borings considered most applicable for the location of each. After the first construction season, the time-rate settlements were recomputed for settlement plates 1, 2, 4, and 8 (fig. 8) using the rate of construction


ELEVATION IN FT MSL
SETTLEMENT IN FT
790
740
690
1
2
3
EL OF FILL ABOVE SETTLEMENT PLATE 4
EL OF FILL ABOVE SETTLEMENT PLATE 1
RECOMPUTED TIME-RATE SETTLEMENT AT PLATE 1
OBSERVED SETTLEMENT AT PLATE 1
RECOMPUTED TIME-RATE SETTLEMENT AT PLATE 4
OBSERVED SETTLEMENT AT PLATE 4
ULTIMATE PREDICTED SETTLEMENT AT PLATE 4
LEGEND
Δ PLATE 1
O PLATE 4
STATION 7+00
EL OF FILL ABOVE SETTLEMENT PLATE 8
EL OF FILL ABOVE SETTLEMENT PLATE 2
RECOMPUTED TIME-RATE SETTLEMENT AT PLATE 2
OBSERVED SETTLEMENT AT PLATE 2
OBSERVED SETTLEMENT AT PLATE 8
RECOMPUTED TIME-RATE SETTLEMENT AT PLATE 8
LEGEND
Δ PLATE 2
O PLATE 8
O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M
1948
1949
1950
1951
STATION 5+50

NOTE: RECOMPUTED TIME-RATE SETTLEMENT CURVES SHOWN ARE BASED ON ACTUAL RATE OF CONSTRUCTION DURING FIRST WORKING SEASON.

Fig. 8. Elevation of fill and predicted and observed settlements for plates 1, 2, 4, and 8

accomplished during the first construction season. The results of these latter computations are the only ones available, and will be discussed together with the observed settlements in paragraph 55. Computed settlement for plate 1 was based on consolidation tests performed on specimens from boring DU-8, for plate 2 on specimens from boring DU-6, for plate 4 on specimens from borings DU-1, -7, and -8, and for plate 8 on specimens from boring DU-3.

20. Seepage. The underseepage through the overburden foundation material was computed using a solution developed by Terzaghi for conditions which assume an impervious weir on a pervious foundation overlying an impervious bedrock when the base width of the weir is greater than the depth of the overburden. The computed quantity is expressed by the following equation:

$$Q = \frac{hk}{0.88 + \frac{2b}{T}}$$

The coefficients used for the foundation were selected from permeability tests performed on samples from test pit 2 (see table 2). The computations were not available for this review so the values of the coefficient of permeability, etc., which were used cannot be cited. Neither can the computed amount of seepage through the foundation be cited, since only the total computed amounts for both the embankment and foundation are available. These amounts are 14, 28, and 41 gpm for heads of 25, 50, and 75 ft, respectively, or about 0.55 gpm per ft of head.

21. Seepage tests were not performed on samples of the jointed abutment rock, but it was decided to excavate a cutoff trench to unweathered rock on both abutment slopes and to grout to such depths as necessary to seal the jointed rock. Grout holes were to have a minimum diameter of 1-1/2 in. and were to be drilled approximately normal to the slopes. The grout was to be pumped with a pressure of 1 psi per foot of hole and was to be of the most effective consistency for the different conditions encountered.

Embankment

Borrow areas and materials

22. Borrow area B, shown at extreme left in fig. 1, was established

as a source for embankment material as a result of the field exploration program. The materials were predominately clay silts (CL-ML), containing about 30 per cent clay sizes (<0.002 mm), and were relatively impervious. Another area which was designated as A (not shown in fig. 1), contained similar material but at a lower elevation than that in area B and this material was subsequently determined to be too wet for use as embankment fill without requiring excessive drying during placing. Natural water contents for materials in area A are not known; those for materials in borrow area B ranged from 18 to 57 per cent and corresponding plastic limits ranged from 18 to 28, respectively.

23. Riprap to be placed on the embankment slopes, as shown in fig. 11 on page 21, was to be obtained locally upstream from the dam site. Filter materials to be placed beneath the riprap protection on the slopes and beneath the downstream portion of the embankment were not available locally and were to be obtained from commercial sources.

Laboratory tests

24. Laboratory tests were performed on one composite mixture from each borrow area which was assumed to be representative of the materials from that area. Mechanical analyses, plastic limit, static compaction, permeability, and consolidated-drained direct shear tests were performed on the representative samples. Water content tests were not performed because of the age of the samples which had been obtained about six years before being tested. Gradation curves are shown in fig. 9 and results of the laboratory tests (except the static compaction tests) are shown in table 3. Subsequent to the initial testing program but prior to construction, additional samples were obtained from borrow area B and tested for natural water content and plastic limit. The results of forty such tests indicated that the natural water content on the average was about 5 per cent (in per cent dry weight) greater than the plastic limit.

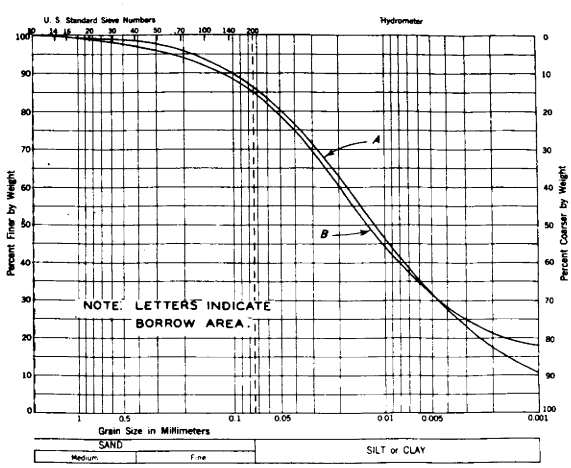


Fig. 9. Gradation of composite borrow materials

TABLE 3. SUMMARY OF TEST DATA ON BORROW MATERIALS

BORING	SAMPLE *	SAMPLE DEPTH OR EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS				ATTER-BERG LIMITS		SP GR	NAT w %	NAT DRY DENSITY LB/CU FT	COMPACTION		SHEAR DATA											PERMEABILITY		CONSOLIDATION DATA				REMARKS
				GVL %	SAND %	FINES ** %	D_{10} MM	LL	PL				OPT w %	MAX DRY DENSITY LB/CU FT	INI-TIAL e	DRY DENSITY LB/CU FT	w_i %	w_f %	S_i %	TYPE TEST†	SPECIMEN SIZE IN.	NO.	σ_{III} TONS/SQ FT	σ_1 TONS/SQ FT	c TONS/SQ FT	ϕ DEG	LOAD TONS/SQ FT	k CM/SEC X 10^{-9}	P_o TONS/SQ FT	P_c TONS/SQ FT	C_c	t_{50} MIN	
	Borrow Area A			1	16	83	0.0009	—	23									22		CD-DS	3-1/4 x 3/4				0.04	29.3	0.508	16					
																					3-1/4 x 3/4						5.075	6					
	Borrow Area B			1	16	83	0.0004	—	22									20		CD-DS	3-1/4 x 3/4				0.04	32.0	0.508	15					
																					3-1/4 x 3/4						5.075	7					

* Composite representative mixtures.

** MIT classification: gravel >2.0 mm, sand 2.0 to 0.06 mm, fines <0.06 mm.

† Specimen sheared at rate of 0.001 in. per min.

CD-DS Consolidated drained-direct shear.

WES FORM NO. 956
OCTOBER 1956

25. Results of the consolidated-drained direct shear tests performed on specimens remolded by application of static loads are shown in fig. 10, together with the selected design shear strength value. The density to which the specimens were compacted is not known. The tests indicated that the materials from the two areas had similar shear strength values and an average of the two, $\phi = 30^{\circ}$, c = 0.04 ton per sq ft, was selected as the design value.

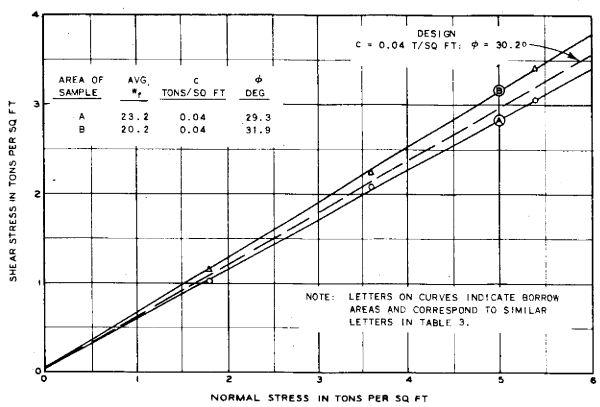

DESIGN
c = 0.04 T/SQ FT; φ = 30.2°
AREA OF SAMPLE
AVG γd
c TONS/SQ FT
φ DEG
A 23.2 0.04 29.3
B 20.2 0.04 31.9
NOTE: LETTERS ON CURVES INDICATE BORROW AREAS AND CORRESPOND TO SIMILAR LETTERS IN TABLE 3.
SHEAR STRESS IN TONS PER SQ FT
NORMAL STRESS IN TONS PER SQ FT

Fig. 10. Consolidated-drained direct shear strength curves for borrow materials

26. Results of the permeability tests are also given in table 3. The design value for the embankment fill material of 11 x 10^{-9} cm per sec is an average of the test results.

Design analyses

27. Section analyzed. The embankment section analyzed, shown in fig. 11, together with a typical as-built section, had a crown width of 20 ft and side slopes of 1 on 4 from natural ground to el 705, 1 on 3-1/2 from el 705 to el 725, 1 on 3 from el 725 to el 745, and 1 on 2-1/2 from el 745 to top of the dam. The downstream slope had 10-ft berms at el 725 and 745. Originally, a 6-ft-thick drainage blanket was to be beneath the downstream portion of the embankment; the thickness of this drainage was subsequently reduced to 3 ft.

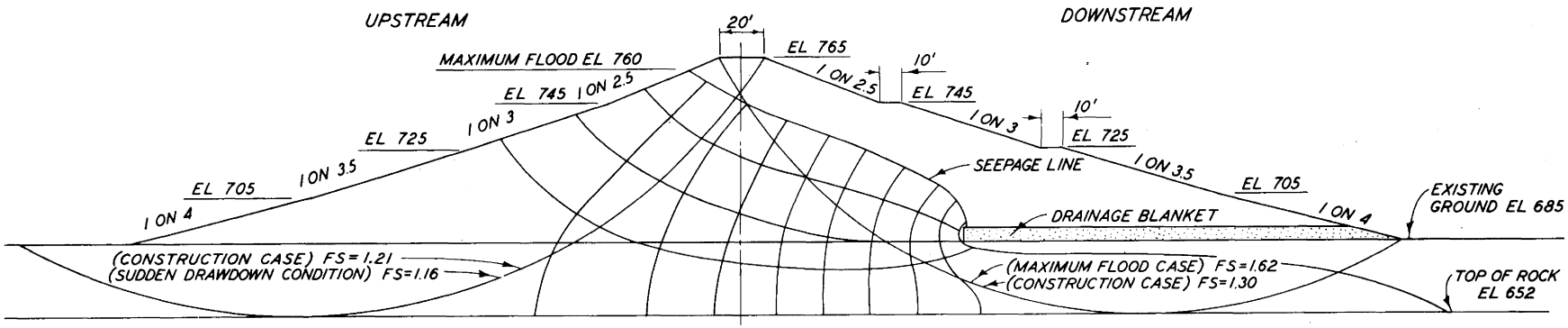
28. Stability. The embankment stability was analyzed by the circular arc method for conditions immediately after construction, reservoir at maximum pool el 760, and for sudden drawdown from maximum reservoir pool. The design constants used are shown in table 4.

Table 4

Soils Design Constants

Material	ϕ°	c ton/sq ft	k cm/sec x 10^{-9}	γ_m lb/cu ft	γ' lb/cu ft
Embankment	30	0.04	11	125	62.5
Foundation	29	0.05	*	125	62.5

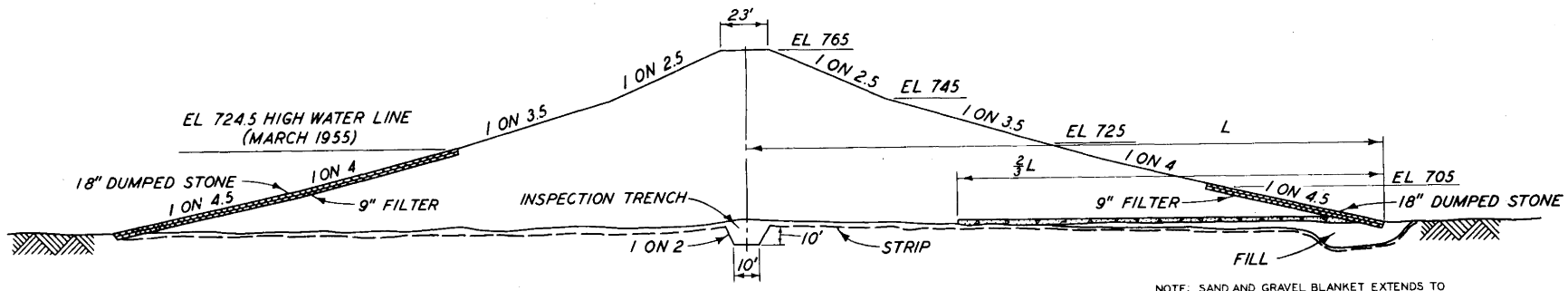
* Values given in table 2 were used for portions of the foundation where they were considered applicable.



TYPICAL DESIGN SECTION WITH CRITICAL FAILURE ARCS AND FLOW NET

STABILITY DATA

SLOPE	CASE ANALYZED	SHEAR STRENGTHS EMBANKMENT DEGREES	EMBANKMENT T/SQ FT	FOUNDATION DEGREES	FOUNDATION T/SQ FT	FOUNDATION PORE PRESSURE IN PER CENT	FACTOR OF SAFETY
UPSTREAM	IMMEDIATELY AFTER CONSTRUCTION	30	0.04	28.8	0.05	65	1.21
	SUDDEN DRAWDOWN FROM MAXIMUM FLOOD	30	0.04	28.8	0.05	0	1.16
DOWNSTREAM	IMMEDIATELY AFTER CONSTRUCTION	30	0.04	28.8	0.05	65	1.30
	MAXIMUM FLOOD	30	0.04	28.8	0.05	—	1.62



TYPICAL AS-BUILT SECTION

NOTE: SAND AND GRAVEL BLANKET EXTENDS TO EL 693 ON EACH ABUTMENT.

Fig. 11. Typical embankment sections

29. Both upstream and downstream slopes were analyzed for the condition immediately after construction. For this condition, it was assumed that the foundation overburden materials would have consolidated only 35 per cent for the embankment load. The effect of this partial consolidation of the foundation was taken into account in the analyses by assuming that only a portion of the embankment load contributed to intergranular stresses along the failure arc in the foundation. The minimum computed factors of safety were 1.21 and 1.30 for the upstream and downstream slopes, respectively.

30. The downstream slope was analyzed for the condition of the reservoir at the maximum flood pool (el 760). Uplift forces acting on the failure arc were determined by use of the flow net shown in fig. 11. The minimum factor of safety for this condition was computed to be 1.62. The condition of sudden drawdown from maximum flood pool reduced the computed factor of safety for the upstream slope to 1.16.

31. A section of the embankment showing the most critical arcs for the conditions discussed above is shown in fig. 11 together with the design constants used and the computed factors of safety. The most critical arc for each of the conditions analyzed was found to go through the foundation overburden to the underlying bedrock. It may be seen in fig. 11 that the typical as-built section of the embankment differs from the section for which the stability analyses were computed. The major difference is the elimination of the 10-ft berms and flattening of the upstream and downstream slopes on the as-built section. These changes increased the base width of the typical section only slightly and the effect on the stability of the section was considered negligible; therefore additional analyses were not performed for the constructed section.

32. Consolidation. Consolidation tests were not performed on compacted borrow materials and an estimate of the amount of embankment consolidation that would occur was not made. The crown elevation of 765 was specified as the net grade at the time of acceptance. The amount of consolidation within the embankment and foundation that would occur subsequent to acceptance was to determine the final net grade.

33. Seepage. Seepage computations were made using heads of 25, 50, and 75 ft. The seepage through the embankment was computed by a method

offered by Gilboy,* assuming a homogeneous embankment on an impervious foundation. The coefficient of permeability used for the embankment, 11 x 10^{-9} cm per sec, was an average of the permeability test results performed on laboratory-compacted borrow materials shown in table 3. As previously mentioned (paragraph 20), the computations of the seepage analyses are not available and the quantity estimated to come through the embankment alone cannot be cited.

Drainage facilities

34. Final design provided for a two-layer, 3-ft-thick sand and gravel blanket to intercept foundation and embankment seepage. The blanket was to be placed on the prepared foundation of the downstream third of the embankment between the abutments to el 693. The bottom 6 in. of the blanket was to be sand and the upper 2-1/2 ft, gravel. The limits for the specified gradation of the filter materials are shown in fig. 12; the fine material is for the lower 6-in. layer and the coarser material for the upper 2-1/2-ft layer.

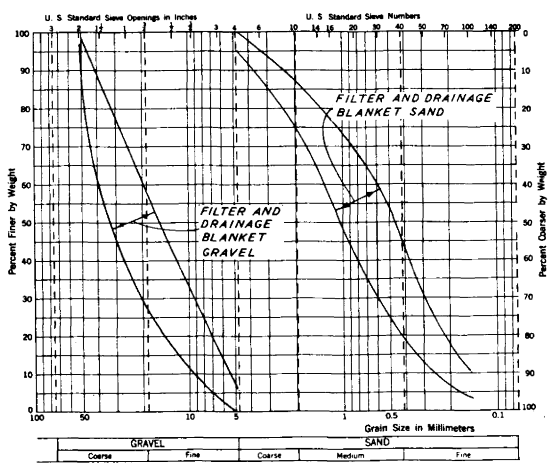


Fig. 12. Gradation bands for filter and drainage blanket sand and gravel

Slope protection

35. The upstream and downstream slopes were to be protected by an 18-in. layer of dumped riprap from the natural ground to el 726 and 708, respectively. The slopes above the dumped stone were to be protected by sod. The riprap was to be underlain by a two-layer filter blanket, the bottom 6 in. of which was to be sand and the upper 3 in., gravel. The stone was to be obtained from sandstone deposits at the dam site and was required to be free from cracks, seams, and other defects that would tend to unduly increase its deterioration from natural causes. The stone was specified to be reasonably well graded with 75 per cent weighing between

* Glennon Gilboy, "Mechanics of hydraulic-fill dams," Proceedings, Boston Society of Civil Engineers (July 1934).

100 and 200 lb and of such shape that neither the thickness nor breadth of any piece would be less than one-third its length, and 25 per cent being spall, weighing less than 50 lb each. The spall was to be suitably shaped for filling voids between the larger stone. The sand and gravel for the riprap filter were to be obtained from commercial sources and were to conform to the same requirements as for the previously discussed sand and gravel drainage blanket materials (see fig. 12).

Spillway and Outlet Works

36. The spillway was to consist of a flat concrete weir keyed into firm rock and was to have fairly short side walls at each end to prevent scouring of the material overlying the bedrock. The outlet works conduit was to be tunneled through the rock and lined with reinforced concrete. Locations of both structures are shown in plan in fig. 1. Since foundations for the spillway and outlet works are rock, detailed discussions of these structures are not pertinent to this report.

PART III: CONSTRUCTION

Borrow Areas and Materials

Impervious soil

37. Impervious soil for the embankment was obtained from borrow area B and the excavations for the diversion channel and inspection trench. Water content of the soil obtained from the low areas of borrow area B was generally greater than that desired for most efficient compaction. However, as will be discussed in detail in the section on embankment construction, the wet materials were used by placing them in alternate layers with drier material obtained from higher and drier areas. It was believed that the dry layers would absorb the extra moisture from the wet layers, resulting in an embankment of uniform, approximately optimum water content throughout.

Pervious material

38. Materials suitable for the filter blanket beneath the dumped riprap and for the sand and gravel drainage blanket beneath the downstream slope were not locally available and were obtained from commercial sources about 15 miles from the dam site. Mechanical analyses were performed on samples of the materials at the time they were delivered to assure that they met filter requirements but results of record tests for gradation of filter material are not available.

39. Riprap considered suitable for placing on the upstream slope from the toe to el 726 was obtained from a local sandstone deposit about 1500-1900 ft upstream from the dam. Riprap placed on the downstream slope from the toe to el 708 was obtained from a commercial limestone quarry. Representative gradations of these are not available, but tests (visual count) were performed at the dam site to ascertain that specified gradations (paragraph 35) were met.

Embankment

Preparation of foundation

40. Initial construction work was begun in September 1948, and consisted of clearing, grubbing, and stripping the foundation and excavating an inspection trench along the axis of the dam. Grubbing consisted of

removing all stumps, roots, and logs larger than 1-1/2 in. in diameter to a depth of 6 ft below the natural ground surface. Stripping consisted of removing the top 6 to 12 in. of topsoil in the flood plain area, 5 to 8 ft in the channel section, and generally something less than 6 in. on the abutment slopes. The inspection trench was dug to a depth of 10 ft (see fig. 2) and revealed no pipe, farm tile, or other utilities.

Stream diversion
and channel filling

41. A diversion channel was excavated parallel to the left abutment and the stream was diverted into it by means of a small dike built in the natural channel above the upstream toe of the dam. The top 5 to 8 ft of the material excavated from the channel was not suitable for fill and was wasted. The first stage of construction was then begun on the right portion of the embankment. The suitable materials excavated from the inspection trench and diversion channel were used for embankment fill and for filling the original stream channel to natural ground surface. Specifications required that all construction be performed in areas free from water. However, it was thought not to be necessary to completely dewater the stream channel since filling operations were started at the upstream toe and progressed downstream, pushing the water ahead of the fill material.

Grouting

42. The abutments were grouted concurrently with the embankment construction with the right abutment being grouted first. Two-inch grout holes were drilled in the inspection trench on the abutment slopes on 10-ft centers to depths shown in fig. 2. The drilling and grouting were started at the bottom of the abutment slopes and advanced upslope although the specifications stated that grouting should progress from the top to the bottom of the abutment slopes. Apparently this change in grouting procedure was permitted so that fill materials could be placed concurrently with the grouting operations. Construction records indicate that the rock of both abutments was relatively tight and that very little grout was taken after the initial holes were completed. Pumping with a bypass pressure pump was continued in each hole until refusal at a pressure of 1 psi per foot of hole. A total of 3200 ft of grout holes was drilled and 4800 cu ft of grout was used.

Placement of sand and gravel drainage blanket

43. Although specifications required a two-layer drainage blanket with the bottom 6 in. of sand and the top 2-1/2 ft of gravel, this was changed during construction and a single-layer, 3-ft sand and gravel blanket was placed on the prepared foundation as shown in fig. 11. The gradation is not known; however, construction records indicate that it was believed to have satisfied filter material requirements.

Compaction control criteria

44. Prior to the beginning of construction, the impervious borrow materials were visually classified into four general groups of soils. Standard compaction curves (B-1, -2, -3, and -4) were prepared for each type soil using standard effort, and these curves were used in the field compaction control program to determine if the desired 95 per cent of standard density was being obtained. These control curves are shown in fig. 13. Optimum water contents for soil types B-1, -2, -3, and -4 were

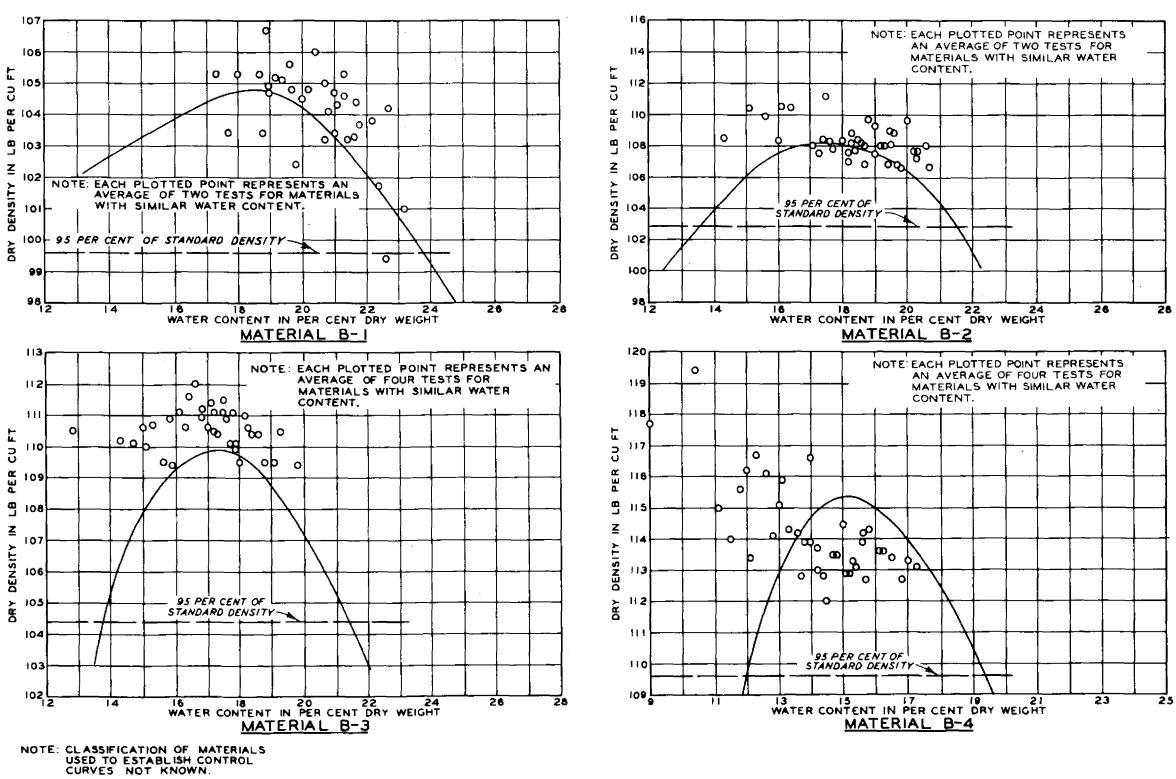

DRY DENSITY IN LB PER CU FT
NOTE: EACH PLOTTED POINT REPRESENTS AN AVERAGE OF TWO TESTS FOR MATERIALS WITH SIMILAR WATER CONTENT.
95 PER CENT OF STANDARD DENSITY
WATER CONTENT IN PER CENT DRY WEIGHT
MATERIAL B-1
NOTE: EACH PLOTTED POINT REPRESENTS AN AVERAGE OF TWO TESTS FOR MATERIALS WITH SIMILAR WATER CONTENT.
95 PER CENT OF STANDARD DENSITY
WATER CONTENT IN PER CENT DRY WEIGHT
MATERIAL B-2
NOTE: EACH PLOTTED POINT REPRESENTS AN AVERAGE OF FOUR TESTS FOR MATERIALS WITH SIMILAR WATER CONTENT.
95 PER CENT OF STANDARD DENSITY
WATER CONTENT IN PER CENT DRY WEIGHT
MATERIAL B-3
NOTE: EACH PLOTTED POINT REPRESENTS AN AVERAGE OF FOUR TESTS FOR MATERIALS WITH SIMILAR WATER CONTENT.
95 PER CENT OF STANDARD DENSITY
WATER CONTENT IN PER CENT DRY WEIGHT
MATERIAL B-4
NOTE: CLASSIFICATION OF MATERIALS USED TO ESTABLISH CONTROL CURVES NOT KNOWN.

Fig. 13. Field control curves and test results

18.6, 17.6, 17.2, and 15.1 per cent of dry weight and dry densities were 104.8, 108.2, 109.9, and 115.4 lb per cu ft, respectively.

Embankment construction

45. The major portion of the embankment fill material was obtained from borrow area B, and the remainder from the excavations for the diversion channel and inspection trench. The embankment specifications stated that the first lift of the embankment should be 9 in. thick (loose) and placed at optimum water content on the foundation which had been thoroughly plowed to a depth of 6 in.; if this lift was too wet after spreading, rolling was to be delayed until the material had dried to a water content that would permit compaction to 95 per cent of standard density. However, after the foundation had been stripped and grubbed, the surface was sufficiently broken up so that additional plowing was considered not necessary and a 2-1/2-ft layer of relatively dry embankment fill was placed and compacted with sheepsfoot rollers. Construction records indicate that this was done to blot up the excess water in the foundation and to form a sufficiently stable layer to permit unrestricted movement of the hauling equipment.

46. Above the initial 2-1/2-ft layer, the embankment was built with alternate layers of wet and dry materials. Each layer was compacted with 9 to 15 passes of sheepsfoot rollers that exerted foot pressures of 355 psi; the foot area of the rollers is not known. The rollers were drawn by crawler-type tractors at approximately 2-1/2 mph. The alternate wet and dry layers were placed and compacted with the 9-15 passes of the sheepsfoot rollers in preference to drying the wet materials and wetting the dry materials and compacting each with a minimum of 6 passes of the sheepsfoot rollers as required by the specifications. Although test data are not sufficiently detailed to support a suggestion that the embankment is not homogeneous, it is believed that the procedure used would quite likely result in a nonhomogeneous structure.

Rate of construction

47. As stated earlier, the embankment from sta 5+00 to the right abutment was constructed in two working seasons to allow anticipated pore pressures in this portion of the foundation to partially dissipate before the full embankment load was applied. In the first season, from October to December 1948, the embankment was constructed to approximately el 730

in about 65 days (see figs. 2 and 8). About 200 days were allowed to pass before additional fill was added and the embankment brought to the final grade of el 765. This second construction season began in May and ended in September 1949.

Compaction control test results

48. A total of 447 field compaction control samples were taken of the embankment (one for approximately every 1000 cu yd of materials placed) and visually classified as one of the four established soil types before determining the dry density and water content. The depth of the samples ranged about 1 to 2 ft below the fill surface. The method of obtaining the field samples and the percentage of the total number of samples that came from the wet or dry layers are not known. Results of the field control tests are plotted with the control curves in fig. 13. The average of the densities of the four soil types ranged from 99 to 101 per cent of standard density and corresponding water contents ranged from 1.8 per cent wet to 1.0 per cent dry of optimum. Fig. 14 shows the variation from optimum

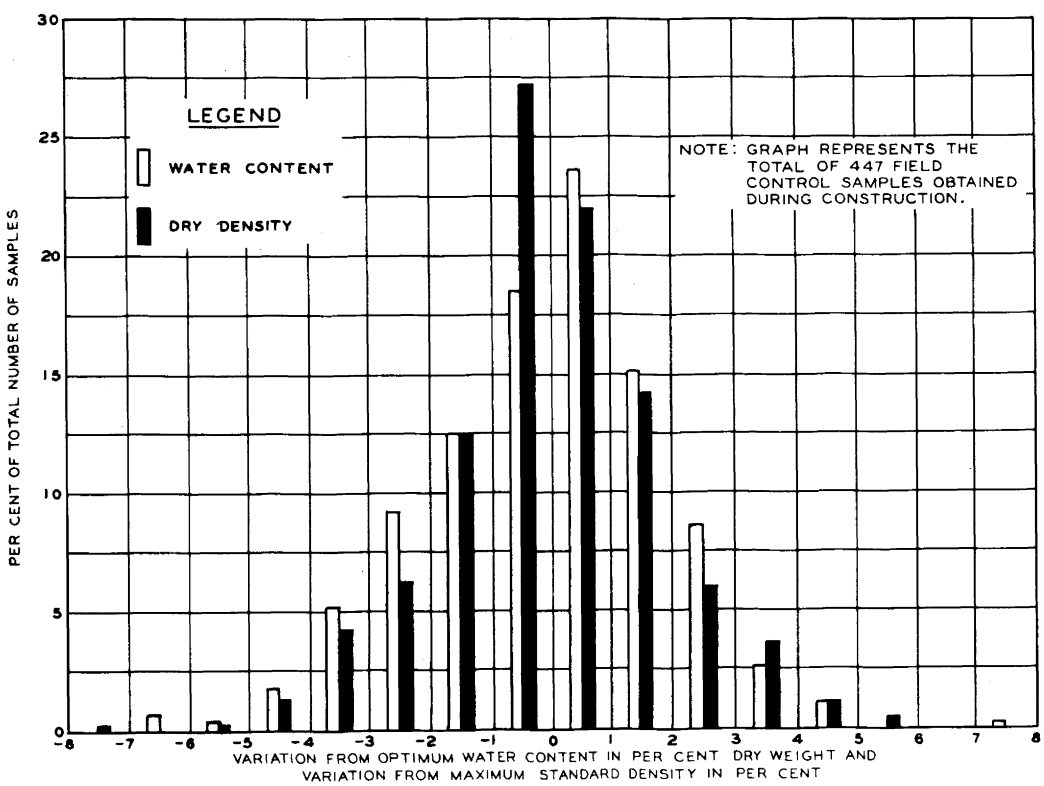

LEGEND
WATER CONTENT
DRY DENSITY
NOTE: GRAPH REPRESENTS THE TOTAL OF 447 FIELD CONTROL SAMPLES OBTAINED DURING CONSTRUCTION.
PER CENT OF TOTAL NUMBER OF SAMPLES
0
5
10
15
20
25
30
-8 -7 -6 -5 -4 -3 -2 -1 0 1 2 3 4 5 6 7 8
VARIATION FROM OPTIMUM WATER CONTENT IN PER CENT DRY WEIGHT AND VARIATION FROM MAXIMUM STANDARD DENSITY IN PER CENT

Fig. 14. Variation from optimum water content and maximum standard density for field control tests

water contents in per cent of dry weights and the variation from standard densities in per cent versus the per cent of the total number of field control samples tested. Water contents generally ranged from 5 per cent wet to 5 per cent dry of optimum, and densities as indicated by the field control samples generally ranged from 95 to 105 per cent of standard density. The average results of the field control tests together with the control values are shown in table 5.

Table 5

Summary of Field Compaction Control Tests

Soil Type	Standard Compaction Effort: Opt w, %	Standard Compaction Effort: γ_d lb/cu ft	Field Control Tests: No. Tests	Field Control Tests: Avg w, %	Field Control Tests: Avg γ_d lb/cu ft	Field Control Tests: Variation* from Opt w, %	Field Control Tests: Per cent of Standard γ_d
1	18.6	104.8	66	20.4	104.2	+1.8	99.5
2	17.6	108.2	80	18.4	108.3	+0.8	100.0
3	17.2	109.9	145	17.0	110.5	-0.2	101.0
4	15.1	115.4	156	14.1	114.2	-1.0	99.0

* + indicates wet of optimum water content.
\- indicates dry of optimum water content.

Record sample test results.

49. Six undisturbed samples were obtained from the embankment during construction for record purposes, and mechanical analyses, water content, density, standard compaction, Atterberg limits, consolidation, permeability, and consolidated-drained direct shear tests were performed. The number of samples obtained was probably sufficient to indicate representative conditions of the embankment. The grain-size curves are shown in fig. 15 and test results are summarized in table 6.

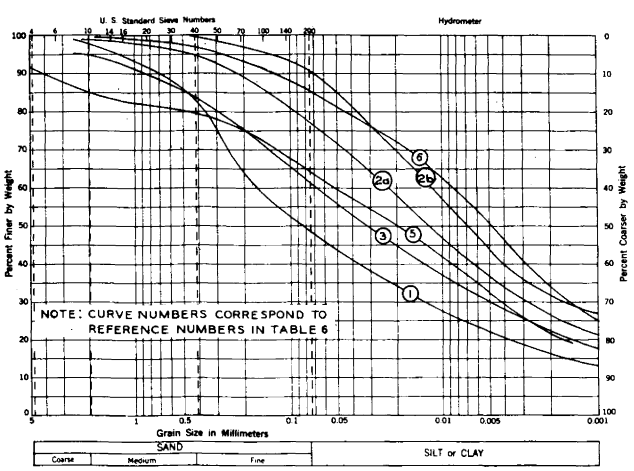


Fig. 15. Gradation of record samples

50. The density and standard compaction tests indicate that the densities of the record samples ranged from 89 to 101 per cent of standard densities with an average of 94 per cent. This is considerably

TABLE 6. SUMMARY OF TEST DATA FOR EMBANKMENT RECORD SAMPLE

REF NO.	STA-TION	SAMPLE DEPTH OR EL	LABORATORY CLASSIFICATION	MECHANICAL ANALYSIS*				ATTER-BERG LIMITS		SP GR	NAT w %	NAT DRY DENSITY LB/CU FT	COMPACTION‡		SHEAR DATA											PERMEABILITY		CONSOLIDATION DATA				REMARKS
				GVL %	SAND %	FINES %	D_{10} MM	LL	PL				OPT w %	MAX DRY DENSITY LB/CU FT	INI-TIAL e	DRY DENSITY LB/CU FT	w_i %	w_f %	S_i %	TYPE TEST**	SPECIMEN SIZE IN.	SPECIMEN NO.	σ_{III} TONS/SQ FT	σ_I TONS/SQ FT	c TONS/SQ FT	ϕ DEG	LOAD TONS/SQ FT	k CM/SEC X 10^{-9}	P_o TONS/SQ FT	P_c TONS/SQ FT	C_c	t_{50} MIN	
1	3+35	692.5		3	51	46		37.9	19.2		15.2	109.1	15.0	112.3													0.25	190		1.8			F.C. br. sand,
																											4.00	26					some silt,
																																	little clay
2a	4+00	694		1	24	75		38.3	23.8		18.8	107.2	15.6	111.2		107.2		19.9		CD-DS	3-1/4 x				0.37	14.0	0.25	140		1.2			Top of sample
																					3-1/4 x						4.0	3.4					
																					3/4												
2b	4+00	694		--	12	88		----	----		22.3	99.1	15.6	111.2		99.1		23.8		CD-DS	3-1/4 x				0.20	21.0	0.25	98		1.4			Bottom of
																					3-1/4 x						4.1	6.7					sample
																					3/4												
3	4+00	697		5	37	58		30.3	19.4		15.5	114.5	13.6	113.1		114.5											0.25	47		1.35			F.C. br. sand
																											4.00	59					and silt,
																																	little clay
4	7+60	703		3	65	32		28.5	20.2		16.1	107.1	12.6	116.7		107.1		14.7		CD-DS	3-1/4 x				0.22	32.0	0.5	110†					Tests made on
																					3-1/4 x						6.1	43†					material pass-
																					3/4												ing No. 4 sieve
5	5+63	720		15	24	61		----	19.9		15.8	109.9	14.0	116.3		109.9		19.8		CD-DS	3-1/4 x				0.23	27.5	0.25	64		1.3			Br. silt, some
																					3-1/4 x						4.00	14					F.C. sand, some
																					3/4						0.5	13†					clay
																											6.1	6†					
6	7+50	730		--	18	72		----	22.8		19.5	107.7	13.6	117.0		107.7		17.7		CD-DS	3-1/4 x				0.32	22.0	0.5	10†		0.92			Gr. br. clay
																					3-1/4 x						6.1	3†					and silt,
																					3/4						0.25	11					little F.C.
																											4.00	2					sand

* Based on MIT classification: gravel >2.0 mm, sand 2.0 to 0.06 mm, fines <0.06 mm.
‡ Standard Proctor.
** Specimen sheared at rate of 0.001 in. per min.
† Test performed on remolded specimen.
CD-DS Consolidated drained-direct shear.

WES FORM NO. 956
OCTOBER 1956

lower than the compaction indicated by the field control tests shown in table 5, where the average densities of the four soil types were 100 per cent of standard density. The water content of the record samples ranged from 0.2 to 6.7 per cent wetter than optimum, with an average of 3.3 per cent wet of optimum. This is considerably wetter than indicated by the field control samples discussed above where a range of 5 per cent wet to 5 per cent dry of optimum was indicated with an average of 0.3 per cent wet of optimum. Results of tests performed on undisturbed record samples are generally considered more reliable than those performed on field control samples and therefore it is considered quite likely that the embankment has an average density somewhat less and an average water content somewhat greater than indicated by the field control tests.

51. Results of the six permeability tests performed on the undisturbed record samples indicated an average coefficient of permeability of 53×10^{-9} cm per sec. The average permeability of the record samples is higher than that used in design (11×10^{-9} cm per sec); however, it may be seen in table 3 that the specimens tested for design were loaded with 0.5 and 5.1 tons per sq ft whereas the record samples (table 5) were loaded with only 0.25 and 4.0 tons per sq ft. It is also pointed out that the laboratory samples were compacted by application of static loads to unknown densities. Examination of tables 3 and 5 reveals that the loads on the samples had a considerable effect on the coefficient of permeability in most cases.

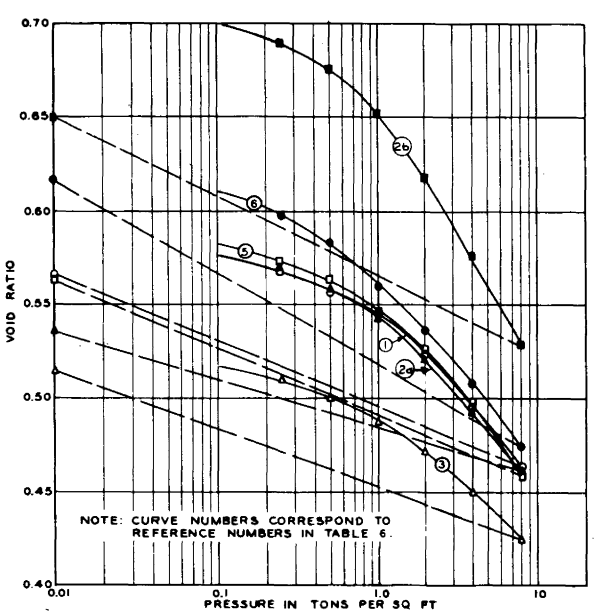


Fig. 16. Consolidation curves for record samples

52. Consolidation tests were performed on the six undisturbed record samples; pressure-void ratio curves are shown in fig. 16. Since no consolidation tests were performed on the laboratory-compacted borrow materials for design purposes, a comparison of the compressibility of laboratory and field-compacted samples is not possible.

53. Consolidated-drained direct shear tests were performed on four undisturbed samples and one remolded sample. Shear strength curves are shown in fig. 17, together with the selected design shear strength curve. The average shear strength value for the tests performed on the undisturbed samples is $\phi = 24^{\circ}$, c = 0.28 ton per sq ft which may be compared with the design strength of $\phi = 30^{\circ}$, c = 0.04 ton per sq ft. It may be noted that for normal stresses less than 2 tons per sq ft, the shearing strengths of the undisturbed record samples are generally greater than design value, but at normal stresses greater than 2 tons per sq ft, they are somewhat less than the design value.

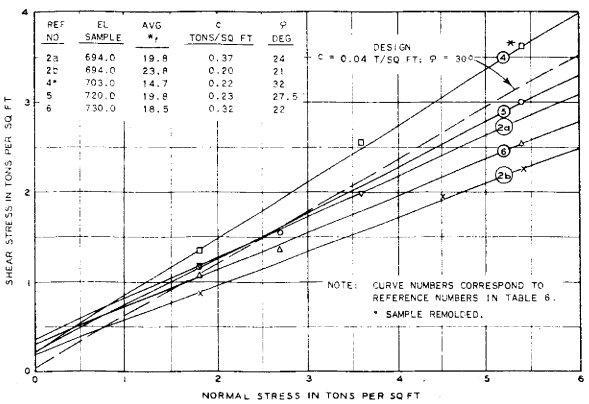


Fig. 17. Consolidated-drained direct shear strength curves for record samples

PART IV: PERFORMANCE OBSERVATIONS

54. Settlement plates were the only permanent measuring devices installed in the embankment. Locations of these are shown in fig. 4 and details are shown in fig. 18.

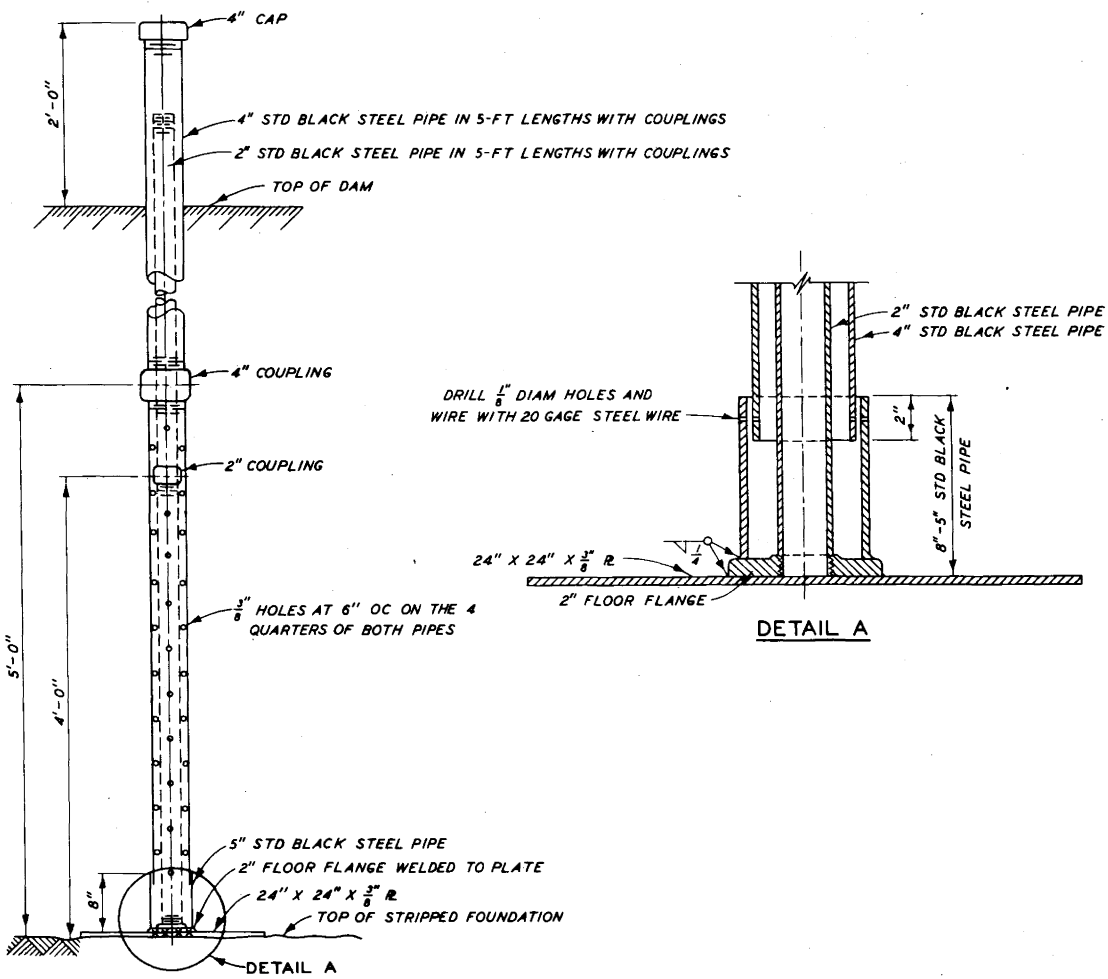

4" CAP
2'-0"
4" STD BLACK STEEL PIPE IN 5-FT LENGTHS WITH COUPLINGS
2" STD BLACK STEEL PIPE IN 5-FT LENGTHS WITH COUPLINGS
TOP OF DAM
4" COUPLING
2" COUPLING
3/8" HOLES AT 6" OC ON THE 4 QUARTERS OF BOTH PIPES
5'-0"
4'-0"
5" STD BLACK STEEL PIPE
2" FLOOR FLANGE WELDED TO PLATE
24" X 24" X 3/8" PL
8"
TOP OF STRIPPED FOUNDATION
DETAIL A
2" STD BLACK STEEL PIPE
4" STD BLACK STEEL PIPE
DRILL 1/8" DIAM HOLES AND WIRE WITH 20 GAGE STEEL WIRE
2"
8"-5" STD BLACK STEEL PIPE
24" X 24" X 3/8" PL
1/4
2" FLOOR FLANGE
DETAIL A

Fig. 18. Settlement plate detail

55. Settlements observed to date are shown in fig. 8 for settlement plates 1, 2, 4, and 8, together with the recomputed settlements for these plates based on the actual rate of construction during the first construction season. Observed settlements for plates 3, 5, 6, 7, and 9 are not shown since computed settlements at these locations are not available for comparison. An examination of fig. 8 indicates that the rate and amount of observed settlement for plates 1 and 8 were very close to those computed.

The observed settlement for plate 2 occurred somewhat faster than the computed rate, but it appears that the total amount of settlement will be close to the computed amount. Observed settlement at plate 4 indicates that much larger settlement occurred and at a much faster rate than predicted. It was believed at the time the observations were made that such a large settlement was partly the result of lateral movement in the foundation rather than consolidation alone.

56. Temporary reference hubs were set on and beyond the toe of the embankment at sta 5+50 and 7+00 to measure possible lateral movement in the vicinity of settlement plate 4. The hubs were installed at the end of the first construction season, and observations were made about 3 months later before additional fill was placed. No significant movements of the hubs were observed at the end of the 3-month period. Whether or not horizontal movement had occurred prior to installation of the reference hubs cannot be determined from recorded data.

57. Based on the rate of the observed settlements, the anticipated pore pressures in the foundation, which dictated the necessity of constructing the right portion of the embankment in two seasons, failed to materialize.

58. No seepage has been observed on the downstream slope of the embankment nor from the drainage blanket at the downstream toe, to date. However, the maximum elevation that the reservoir pool has reached to date is 724.5 in March 1955, or only slightly above maximum conservation pool elevation.